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GEOTECHNICAL INVESTIGATION
Camp Williams
17800 South Camp Williams Rd.
Riverton, Utah

IGES Job No. 01273-002

July 1, 2009

Prepared for:

Stanley Consultants
c/o Jeff Masters

IGES

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APPENDICES

1.0 EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation conducted for the proposed power distribution upgrade building to be located directly north of Building 1000 South near the northwest corner of Camp Williams near the border of Salt Lake and Utah Counties in Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading and the design and construction of foundations, slabs-on-grade, and concrete flatwork.

As a part of this investigation, near surface soil conditions were explored by excavating two test pits at the site. A member of our technical staff visually logged soils in the test pits at the time of excavation in general accordance with the Unified Soil Classification System. Undocumented fill material, approximately the upper 2½ feet of the site was observed during our exploration. The undocumented fill consisted of Lean CLAY (CL) with some gravel. Underlying the fill we encountered Poorly Graded GRAVEL (GP) and alternating layers of SILT (ML), Silty SAND (SM), and Poorly Graded SAND (SP).

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed development provided that the recommendations contained in this report are incorporated into the design and construction of the project. The foundations for the proposed structure may consist of conventional shallow spread footings founded below the undocumented fill and entirely on competent, undisturbed native soil, or structural fill. We recommend that IGES observe the bottom of the foundation excavations prior to the placement of structural fill, steel, or concrete to assess any unsuitable soils. If over-excavation is required, the entire structure should be founded on a minimum of 1 foot of structural fill. All undocumented fill should be removed beneath footings.

NOTE: The scope of services provided within this report are limited to the assessment of the subsurface conditions at the subject site. The executive summary is provided solely for purposes of overview and is not intended to replace the report of which it is part and should not be used separately from the report.

2.0 INTRODUCTION

2.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical investigation conducted for the proposed power distribution upgrade building to be located directly north of Building 1000 South near the northwest corner of Camp Williams near the border of Salt Lake and Utah Counties in Utah. The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the site and to provide recommendations for general site grading and the design and construction of foundations, slabs-on-grade, and concrete flatwork.

The scope of work completed for this study included a site reconnaissance, subsurface exploration, soil sampling, laboratory testing, engineering analyses, and preparation of this report. Our services were performed in accordance with our proposal, dated June 2, 2009 and signed authorization.

The recommendations contained in this report are subject to the limitations presented in the "Limitations" section of this report (Section 7.1).

2.2 PROJECT UNDERSTANDING AND DESCRIPTION

The location of the proposed power distribution upgrade building is located on the Site Vicinity Map (Plate A-1). The proposed location for the building is located directly north of Building 1000, southeast of Beef Hollow, and several hundred feet east of SR-68 (Redwood Rd.). The site is currently undeveloped and covered with gray gravel over fill. The site is relatively flat but slopes slightly to the north.

We understand that construction of this building will consist of a prefab structure with an approximate foot print of 61 by 35 feet. The structure is anticipated to be lightly loaded and require underground utility and instrumentation connections.

3.0 METHODS OF STUDY

3.1 FIELD INVESTIGATION

As a part of this investigation, near surface soil conditions were explored by excavating two test pits at the site. A member of our technical staff visually logged soils in the test pits at the time of excavation in general accordance with the Unified Soil Classification System (USCS). The test pits were excavated to an approximate depth of 11 feet below the existing site grade. Test pit logs are included at the end of this report (Plates A-3 thru A-4) and a *Key to Soil Symbols and Terminology* is also provided as Plate A-5. A discussion of the site conditions is provided in Section 4.0 of this report.

The test pits were excavated with a Case 580 rubber-tired back hoe. Representative soil samples were collected and classified by a member of our technical staff. A single, relatively undisturbed sample was collected with the use of a U-type hand sampler driven by a 2 lb. sledge hammer. Bulk samples and other disturbed samples were collected and placed in buckets and bags. The samples were carefully packaged and transported to our laboratory for testing.

3.2 LABORATORY INVESTIGATION

Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the engineering characteristics of onsite earth materials. Laboratory tests conducted during this investigation include:

- In situ moisture content and dry density
- Atterberg Limits
- No. 200 Sieve Wash
- Grain Size Distribution
- Direct Shear Test (ASTM D3080)
- Water-soluble sulfate concentration for cement type recommendations
- Resistivity and pH to evaluate corrosion potential of ferrous metals in contact with site soils

The results of some laboratory tests are shown on the test pit logs (Appendix A). The results of all laboratory tests are presented on the test result plates presented in Appendix B (Plates B-1 through B-3) and in the *Summary of Laboratory Test Results Table* (Plate B-4).

3.3 ENGINEERING ANALYSIS

Engineering analyses were performed using soil data obtained from the laboratory test results and empirical correlations from material density, depositional characteristics and classification. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care.

4.0 GENERALIZED SITE CONDITIONS

4.1 SURFACE CONDITIONS

At the time of our field investigation the site was undeveloped and covered with a layer of gray gravel. The site is relatively flat but slopes slightly down to the north; this area has been used in the past for the parking of heavy army equipment such as trucks, tanks, and construction equipment.

4.2 SUBSURFACE CONDITIONS

The subsurface soil conditions were explored at the subject property by excavating two test pits at the site. Subsurface soil conditions were logged during our field investigation and are included in the Test Pit Logs in Appendix A at the end of this report (Plates A-3 & A-4). The soil and moisture conditions encountered during our investigation are discussed below.

4.2.1 Earth Materials

Based on our observations and geologic literature review, the site is underlain by Pleistocene-aged sand and gravel lacustrine deposits associated with Provo (regressive) phase of the Lake Bonneville cycle. However, undocumented fill material, approximately the upper 2½ feet of the site was observed during our exploration. The undocumented fill consisted of Lean CLAY (CL) with some gravel.

Underlying the fill we encountered Poorly Graded GRAVEL (GP) and alternating layers of SILT (ML), Silty SAND (SM), and Poorly Graded SAND (SP). In TP-1 the Poorly Graded GRAVEL (GP) underlying the fill extended the entire depth of the test pit. In TP-2, a relatively thin layer of Poorly Graded GRAVEL (GP) was observed beneath the undocumented fill; a relatively thin layer of Sandy SILT (ML) was observed beneath the Poorly Graded GRAVEL (GP).

The stratification lines shown on the enclosed test pit logs represent the approximate boundary between soil types (Plates A-3 & A-4). The actual in-situ transition may be gradual. Due to the nature and depositional characteristics of the native soils, care should be taken in interpolating subsurface conditions between and beyond the exploration locations.

4.2.2 Strength of Earth Materials

A Direct Shear Test (ASTM D3080) was performed on a representative soil sample that classifies as Sandy SILT (ML). The result indicated that the sample tested had an internal friction angle (phi) of 35 degrees with 408 psf cohesion. A summary of the test results are presented in Appendix B (Plate B-3).

4.2.3 Groundwater/Moisture Conditions

Groundwater was not encountered in any of our exploratory test pits for this project. The soil moisture was described as *slightly moist* in all of the test pits. The moisture content generally ranged from 5 to 22% in the native soils observed in the test pits.

Groundwater is not expected to impact construction of the proposed structures. Due to the season of our investigation (late spring), groundwater levels are expected to be near their seasonal low. It is our experience that during snowmelt, runoff, irrigation on surrounding properties, high precipitation events, and other activities, the groundwater level can fluctuate several feet.

5.0 GEOLOGIC CONDITIONS

5.1 GEOLOGIC SETTING

The site is located between the elevations of 4,751 to 4,761 feet in the northern portion of Utah Valley. This valley represents a deep, sediment-filled structural basin of Cenozoic-age flanked by uplifted blocks; the Wasatch Range on the east, and the Lake Mountains, West Mountain, the Goshen Hills, and Warm Springs Mountain (the northern end of Long Ridge) to the west (Machette, 1992; and Hintze, 1980). The Wasatch Range is the easternmost expression of pronounced Basin and Range extension in north-central Utah. The Traverse Mountains, located just north of the site, form a prominent salient along the Wasatch Front that separates the basin into two distinct valleys. This salient is also a structural boundary that divides the Salt Lake City and Provo segments of the Wasatch Fault Zone. The Traverse Mountains are one of the four salients that form structural barriers along the Wasatch Fault Zone (Machette, 1992).

The near-surface geology of the Utah Valley is dominated by sediments, which were deposited within the last 30,000 years by Lake Bonneville (Scott et al., 1983; Hintze, 1993; Machette, 1992). Lake Bonneville was the largest late Pleistocene lake in western North America. The lake covered nearly 20,000 square miles of western Utah and portions of southern Idaho and western Nevada (Gwynn, 1996). At its peak, the lake was approximately 325 miles long, 135 miles wide, and 1,000 feet deep in some areas. The Bonneville Shoreline represents the highest lake level (5,090 feet) and formed between 16,000-14,500 years ago (Hintze, 1993). The Provo Shoreline (4,900 feet) formed between 14,500-12,500 years ago after the Bonneville flood. The Bonneville flood occurred approximately 15,000 years ago when Lake Bonneville crested at Red Rock Pass in Idaho after eroding a natural dam comprised of alluvial fan deposits. Approximately 1,000 cubic miles of water was released causing the lake to drop 300 feet to the Provo Shoreline (U.S.G.S website, 2003). The main conduit for the Bonneville flood was the Snake River. When the flood waters reached the Snake River Canyon, they were approximately 350 feet deep and were flowing at about 70 miles per hour. It is estimated that the flood occurred over a period of a couple weeks until it reached the Provo Level. Once at the Provo level, the lake slowly receded, due to changes in Earth's climate. The climate slowly warmed and became drier, causing the remaining water to evaporate over time. As the lake slowly receded, streams began to incise the large deltas that had formed at the mouths of major canyons along the Wasatch Range; the eroded material was deposited in shallow lakes and marshes in the basin and in a series of recessional deltas and alluvial fans. Sediments toward the center of the valley are predominately deep-water deposits of clay, silt and fine sand. However, these deep-water deposits are in places covered by a thin post-Bonneville alluvial cover.

Surface sediments at the subject site are mapped as upper Pleistocene lacustrine deposits of sand and gravel (Qlgp) related to the Provo (regressive) phase of the Lake Bonneville cycle (Biek, 2005). These sediments were formed as a result of highly fractured bedrock exposed to wave action associated with Lake Bonneville. Strong persistent waves approached from the north-northwest, causing north-south longshore sediment transport (Schofield et al., 2004). These northerly winds were generated by the high atmospheric pressure cell associated with the continent ice sheet (Jewell, 2007). As such, the North American jet stream is believed to have been south of Lake Bonneville as recently as 12,000 years ago (Jewell, 2007).

5.1.1 Geologic Units

Quaternary Lacustrine Deposits – Bonneville Lake Cycle

***Qlgp* (Upper Pleistocene):** Quaternary Lacustrine Deposits – Bonneville Lake Cycle. Moderately to well-sorted, moderately to well-rounded, clast supported gravel and sand; typically thin to thick bedded with some interbedding of beds (Biek, 2005). Also is locally partially cemented with calcium carbonate, contains gastropods in sandy lenses, and forms well-developed wave-cut or wave-built terraces. Thickness varies from 0 to 300 feet.

5.2 SEISMICITY AND FAULTING

An *active* fault is defined as a fault that has moved within the Holocene (approximately 10,000 ybp). There are no known active faults that pass under or immediately adjacent to the site (Black et al., 2003). The Jordan Narrows fault has been inferred to directly lie approximately 0.4 miles west of the site (Biek, 2005). This fault is not considered active because it is not known to offset Quaternary sediments. The site is located approximately 6 miles north of the active Utah Lake faults and folds. These features are poorly understood northeast- to northwest-trending faults and folds located beneath Utah Lake and are reported to have been active in the past 15 ka (Black et al., 2003). These structures were identified from seismic reflection data and appear to be offsetting latest Pleistocene to Holocene sediments. The slip rate is estimated to be 0.2-1mm/yr and dip direction is east and west (Black et al., 2003).

The site is also located approximately 3½ miles west of the Provo segment of the Wasatch Fault

Zone. The Wasatch Fault Zone is a series of normal faults that mark the eastern boundary of the Intermountain Seismic Belt (ISB) through northern and central Utah. The Wasatch Fault Zone is comprised of ten discrete north-south trending segments totaling 342 kilometers in length that extend southward from Malad City, Idaho to Fayette, Utah (Black et al., 2003). Each fault segment is believed to be independent from the other segments. As a result, no fault rupture is likely to occur along all of the segments of the Wasatch Fault Zone during a single event (Hintze, 2005). The Provo segment is one of the longest (70 km) and most active segments of the Wasatch Fault Zone (Machette, 1992). The most recent earthquake to date along the Provo segment of the Wasatch Fault Zone occurred approximately 600 years ago (Hintze, 2005). The Provo segment dip direction varies from 58º to 79ºW and its slip rate is estimated to be 1-5 mm/yr (Black et al., 2003). Analyses of ground shaking hazard along the Wasatch Front suggests that the Wasatch Fault Zone is the single greatest contributor to the seismic hazard in Utah Valley region.

Using the criteria outlined in the 2006 IBC, the maximum considered earthquake (MCE) ground motion is taken as that motion represented by an acceleration response spectrum having a 2% chance of exceedance within a 50-year period (Section 1613.5). This hazard was assessed for the site using the Java Application Ground Motion Parameter Calculator – Version 5.0.9 developed by the USGS located on their website at http://earthquake.usgs.gov/research/hazmaps/design/, which correlates with the International Building Code (2006 IBC) seismic hazard maps. This program, as with the IBC maps, is used to develop the probabilistic spectral accelerations corresponding to MCE seismic hazard level for rock-like conditions. To account for site soil effects, site coefficients (F_a and F_v) were used to attenuate the rock-based spectral acceleration values. Based on our observations of soils at the site, it is our opinion that the soils at this site are representative of a "stiff soil" profile; best described by IBC Site Class D with F_a and F_v values of 1.05 and 1.54, respectively. From these procedures the MCE PGA was established to be 0.47g. The MCE and Design response spectrum are presented on Plate C-1 in Appendix C. The following table presents response accelerations for 0.2 and 1.0 second periods.

MCE Seismic Response Spectrum Spectral Acceleration Values [a]

Site Location: Latitude = 40.4382° N Longitude = -111.9284° W	Site Class D Site Coefficients: F_a = 1.05 F_v = 1.54
Spectral Period (sec)	**Response Spectrum Spectral Acceleration (g)**
0.200	1.179
1.000	0.705

[a] 2006 IBC recommends scaling the MCE values by 2/3 to obtain the design spectral response acceleration values.

5.3 OTHER GEOLOGIC HAZARDS

Geologic hazards can be defined as naturally occurring geologic conditions or processes that could present a danger to human life and property. These hazards must be considered before development of a site. There are several hazards in addition to seismicity and faulting that may be present at the site, and which should be considered in the design of critical facilities such as water tanks and structures designed for human habitation. The other geologic hazards considered for this site are liquefaction and landslide. A complete list of potential geologic hazards is included in the *Summary of Geologic Hazards Table* in Appendix C of this report (Plate C-2).

5.3.1 Liquefaction

Certain areas within the Intermountain region possess a potential for liquefaction during seismic events. Liquefaction is a phenomenon whereby loose, saturated, granular soil deposits lose a significant portion of their shear strength due to excess pore water pressure buildup resulting from dynamic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in densification of such deposits causing settlements of overlying layers after an earthquake as excess pore water pressures are dissipated. The primary factors affecting liquefaction potential of a soil deposit are: (1) level and duration of seismic ground motions; (2) soil type and consistency; and (3) depth to groundwater.

Referring to the map titled "*Surface Rupture Liquefaction Potential Special Study Areas, Salt Lake County*" dated March 2002 and published by the Salt Lake County Public Works Department, the subject site is located within an area currently designated as "very low" for liquefaction potential. “Very low” liquefaction potential means that there is a less than 5% probability of having an earthquake within a 100-year period that will be strong enough to cause liquefaction.

5.3.2 Landslides

There are several types of landslides that should be considered when evaluating geologic hazards at a site. These include shallow debris slides, deep-seated earth or rock slumps and earth flows. These landslide types can be described as being *older*, *younger*, or *historical*. This division is based on the degree to which the characteristic features of these landslides are preserved. *Historical* landslides are characterized by hummocky topography, numerous internal scarps, and chaotic bedding, as well as more recent evidence such as tilted trees, fresh scarps, and damaged roads, utilities, or other structures. The characteristics of *younger* landslides are similar to those of *historic* landslides but do not appear to be as recent. The characteristic features of *older* landslides are morphologically subtle and sometimes indistinguishable.

None of these landslide types are reported at the site, and none were observed during our field investigation. However, several historically active slides have been mapped approximately ½ mile northeast of the site along the Jordan River (Harty, 1992). All of these slides are relatively small and appear to be in lacustrine gravel and sand deposits associated with the Provo Level of Lake Bonneville.

It should be noted that the absence of the geomorphic expression of landslides does not preclude the existence of landslides on the site. Furthermore, it should be noted that the evaluation of landslides in this report is based on literature review and cursory site observations only. *This section addresses potential existing landslides and should not be construed to be an evaluation of on-site slope stability, either surficial or deep-seated, which is a different subject and is beyond the scope of our services*.

6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL CONCLUSIONS

Based on the subsurface conditions encountered at the site, it is our opinion that the subject site is suitable for the proposed development provided that the recommendations contained in this report are incorporated into the design and construction of the project. The foundations for the proposed structure may consist of conventional shallow spread footings founded below the undocumented fill and entirely on competent, undisturbed native soil, or structural fill. We recommend that IGES observe the bottom of the foundation excavations prior to the placement of structural fill, steel, or concrete to assess any unsuitable soils. If over-excavation is required, the entire structure should be founded on a minimum of 1 foot of structural fill. Undocumented fill material (approximately the upper 2½ feet on the site) was observed during our exploration. This fill material was encountered in both test pits. All undocumented fill should be removed beneath footings.

If subsurface conditions other than those described herein are encountered during construction or if design and layout changes are initiated, IGES, Inc. must be informed so that our recommendations can be reviewed and revised as changes or conditions may require.

The following sub-sections present our recommendations for general site grading, design of foundations, slabs-on-grade, lateral earth pressures, and soil corrosion.

6.2 EARTHWORK

Prior to the placement of foundations, general site grading is recommended to provide proper support for foundations, exterior concrete flatwork, and concrete slabs-on-grade. Site grading is also recommended to provide proper drainage and moisture control on the subject property and to aid in preventing differential settlement of foundations as a result of variations in subgrade moisture conditions.

6.2.1 General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all undocumented fill should be removed. Any existing utilities should be re-routed or protected in-place. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment such as a scraper or loader.

Any soft/loose areas identified during proof-rolling should be removed and replaced with structural fill as described in Section 6.2.4 of this report.

6.2.2 Excavations

If soft, loose, or otherwise deleterious earth materials such as undocumented fill are encountered, these soils may require over-excavation and subsequent replacement with structural fill as recommended in Section 6.2.4. If required, the excavations should extend a minimum of 1 foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond slabs-on-grade and pavements.

6.2.3 Excavation Stability

The contractor is responsible for site safety, including all temporary trenches excavated at the site and design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by Occupational Safety and Health Administration (OSHA) standards to evaluate soil conditions. Soil types are expected to consist primarily of Type C soils (sandy soils). Close coordination between the “competent person” and IGES, Inc. should be maintained to facilitate construction while providing safe excavations.

Based on OSHA guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions are encountered, or when the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. Sloping the sides at one and one half horizontal to one vertical (1.5 H:1V) in accordance with OSHA Type C soils may be used as an alternative to shoring or shielding.

6.2.4 Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements, should consist of structural fill. Structural fill may consist of excavated onsite soils having an Expansion Index less than 20. Material not meeting the aforementioned criteria may be suitable for use as structural fill; however, such material should be evaluated on a case-by-case basis and should be approved by IGES prior to use. In all cases, structural fill should be free of vegetation and debris, and contain no rocks larger than 4 inches in nominal size (6 inches in greatest dimension).

All structural fill should be placed in maximum 6-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 8-inch loose lifts if compacted by light-duty rollers, and maximum 12-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill placed beneath footings and pavements should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557. The moisture content should be at or slightly above the OMC for all structural fill. Prior to placing any fill, the excavations should be observed by IGES to assess that unsuitable materials have been removed. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

We recommend that all utility trenches backfilled below pavement sections, curb and gutter and concrete flatwork, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches, including landscape areas, should be backfilled and compacted to approximately 90 percent of the MDD (ASTM D-1557).

Specifications from governing authorities having their own precedence for backfill and compaction should be followed where more stringent.

6.3 FOUNDATIONS

Based on our field observations and laboratory data, the proposed structures may be founded below the undocumented fill and directly upon relatively undisturbed, competent native soils. We recommend that IGES observe the bottom of the foundation excavation prior to the placement of structural fill, steel or concrete to identify any unsuitable soils. If over-excavation is required, the entire structure should be founded on a minimum of 1 foot of structural fill.

If required, all fill beneath the foundations should be placed and compacted in accordance with our recommendations contained in Section 6.2.4 of this report. Shallow spread footings constructed *entirely* on competent relatively undisturbed native soil or *entirely* on a minimum of 1 foot of structural fill over competent relatively undisturbed native soil may be proportioned utilizing a maximum net allowable bearing pressure of **2,500 pounds per square foot (psf)** for dead load plus live load conditions.

All foundations exposed to the full effects of frost should be established at a minimum depth of 30 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost (i.e., a continuously heated structure), may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes. The minimum recommended footing width is 24 inches for continuous wall footings (if any) and 36 inches for isolated spread footings.

6.4 SETTLEMENT

Static settlement of properly designed and constructed conventional foundations, founded as described above, are anticipated to be on the order of 1 inch or less. Differential settlement is expected to be half of total settlement over a distance of 30 feet.

6.5 EARTH PRESSURES AND LATERAL RESISTANCE

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.40 for sandy soils should be used.

Ultimate lateral earth pressures from natural soils and *granular* backfill acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table:

Condition	**Level Backfill**	
	Lateral Pressure Coefficient	**Equivalent Fluid Density (pcf)**
Active (K_a)	0.31	40
At-rest (K_o)	0.5	60
Passive (K_p)	3.3	400

These coefficients and densities assume no buildup of hydrostatic pressures. The force of the water should be added to the presented values if hydrostatic pressures are anticipated. Clayey soils drain poorly and may swell upon wetting, thereby greatly increasing lateral pressures acting on earth retaining structures; therefore, clayey soils should not be used as retaining wall backfill. Backfill should consist of either native granular soil or sandy imported material with an Expansion Index (EI) less than 20.

Walls and structures allowed to rotate slightly should use the active condition. If the element is constrained against rotation, the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

6.6 CONCRETE SLABS-ON-GRADE CONSTRUCTION

To minimize settlement and cracking of slabs, and to aid in drainage beneath the concrete floor slabs, all concrete slabs should be founded on a minimum 6-inch layer of compacted gravel overlying competent native earth materials or structural fill. The gravel should consist of free draining gravel or road base with a 3/4-inch maximum particle size and no more than 5 percent passing the No. 200 mesh sieve. The layer should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557. Other earth materials not meeting the criteria above may be suitable for construction; alternate materials should be evaluated on a case-by-case basis and should be approved by IGES.

All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with a welded wire fabric, re-bar, or fiber-mesh. Slab reinforcement should be designed by the structural engineer. We recommend that concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. If slump and/or air content are beyond the recommendations as specified in the plans and specifications, the concrete may not perform as desired. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI).

A moisture barrier (vapor retarder) consisting of 10-mil thick Visqueen (or equivalent) plastic sheeting should be placed below slabs-on-grade where moisture-sensitive floor coverings or equipment is planned. Prior to placing this moisture barrier, any objects that could puncture it, such as protruding gravel or rocks, should be removed from the building pad. Alternatively, the building pad may be covered by two inches of clean sand.

6.7 MOISTURE PROTECTION AND SURFACE DRAINAGE

It is important that moisture not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the proposed facility should be implemented. We recommend that hand-watering, desert landscaping, or Xeriscape be considered within 5 feet of the foundations. We further recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from structures or to storm water runoff areas. Additionally, the ground surface within 10 feet of the structures should be constructed so as to slope a minimum of five percent away. Alternatively, if the perimeter of the building (within 10 feet of the structure) is covered with a relatively impermeable covering such as asphalt or concrete pavement, a minimum slope of 1% is recommended. Pavement sections should be constructed to divert surface water off of the pavement into storm drains. Parking strips and roadway shoulder areas should be constructed to prevent infiltration of water into the areas surrounding pavement.

6.8 PRELIMINARY SOIL CORROSION POTENTIAL

Laboratory test results indicate that near surface native soils tested have a soluble sulfate content of 41 ppm. Based on this result, the soils are classified as having a low potential for sulfate attack with concrete. We anticipate that conventional Type I/II cement can be used for all of the concrete placed at the site.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil, a representative soil sample was tested in our soils laboratory for soil resistivity (AASHTO T288), soluble chloride content, and pH. The tests indicated that the onsite soil tested has a minimum soil resistivity of 388 OHM-cm, a soluble chloride content of 760 ppm, and a pH of 7.9. Based on this result, the onsite native soil is considered **very corrosive** to ferrous metal. Consideration should be given to retaining the services of a qualified corrosion engineer to provide an assessment of any metal in contact with existing site soils, particularly ancillary water lines, reinforcing steel, and valves.

7.0 CLOSURE

7.1 LIMITATIONS

The recommendations contained in this report are based on our limited field exploration, laboratory testing, and understanding of the proposed construction. The subsurface data used in the preparation of this report were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, we should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, IGES should be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No warranty, expressed or implied, is made.

It is the Client's responsibility to see that all parties to the project including the Designer, Contractor, Subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

7.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction. IGES staff should be on site to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Observation of foundation soils to assess their suitability for footing placement.
- Observation of soft/loose soils over-excavation.
- Observation of temporary excavations and shoring.
- Consultation as may be required during construction.
- Quality control and observation of concrete placement.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 748-4044.

8.0 REFERENCES CITED

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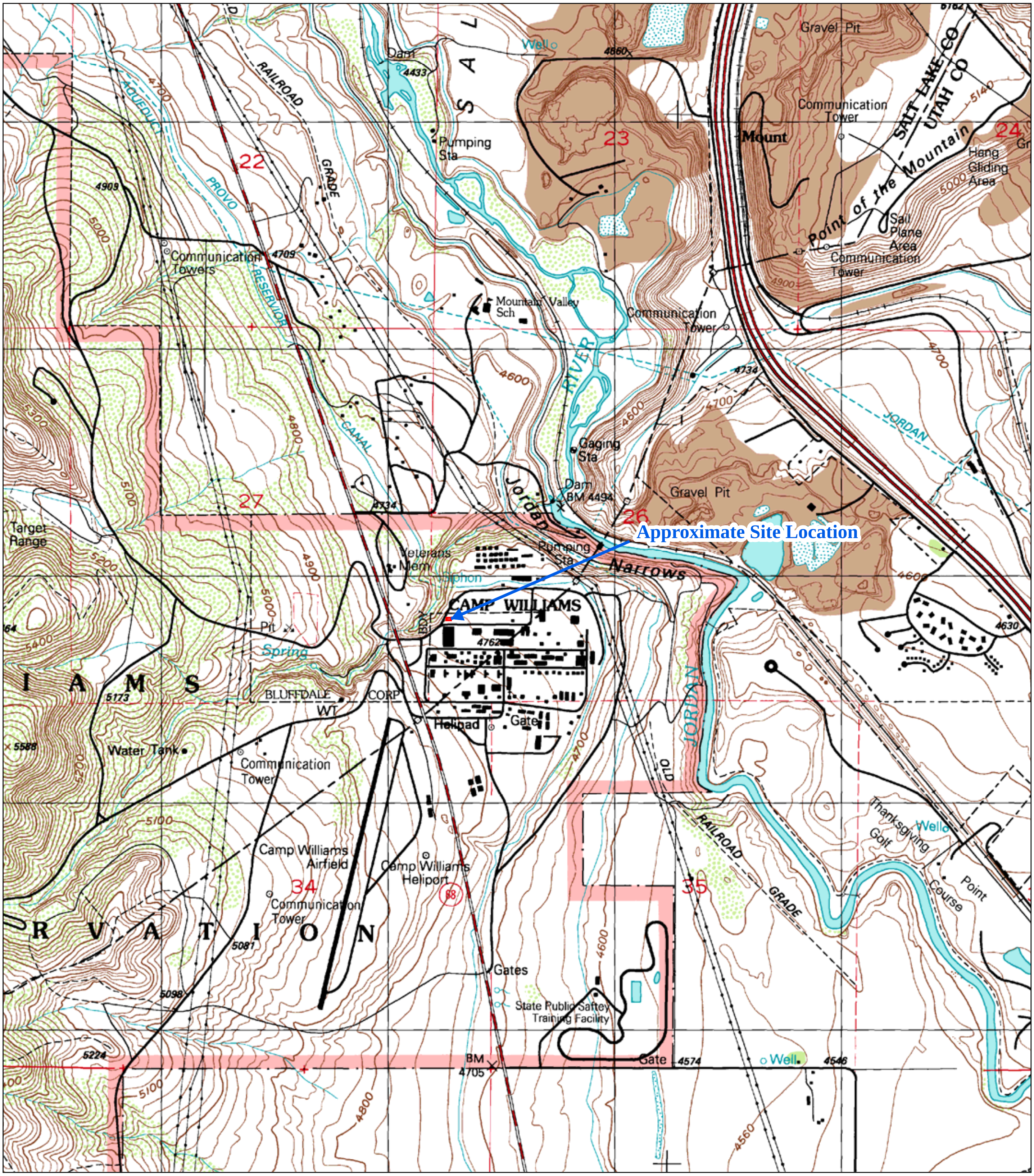
APPENDIX A

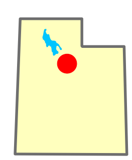


BASE MAP:
JORDAN NARROWS, UTAH
U.S.G.S. 7.5 MINUTE QUADRANGLES

0 | 1,350 | 2,700 | 5,400 Feet

1:24,000

N



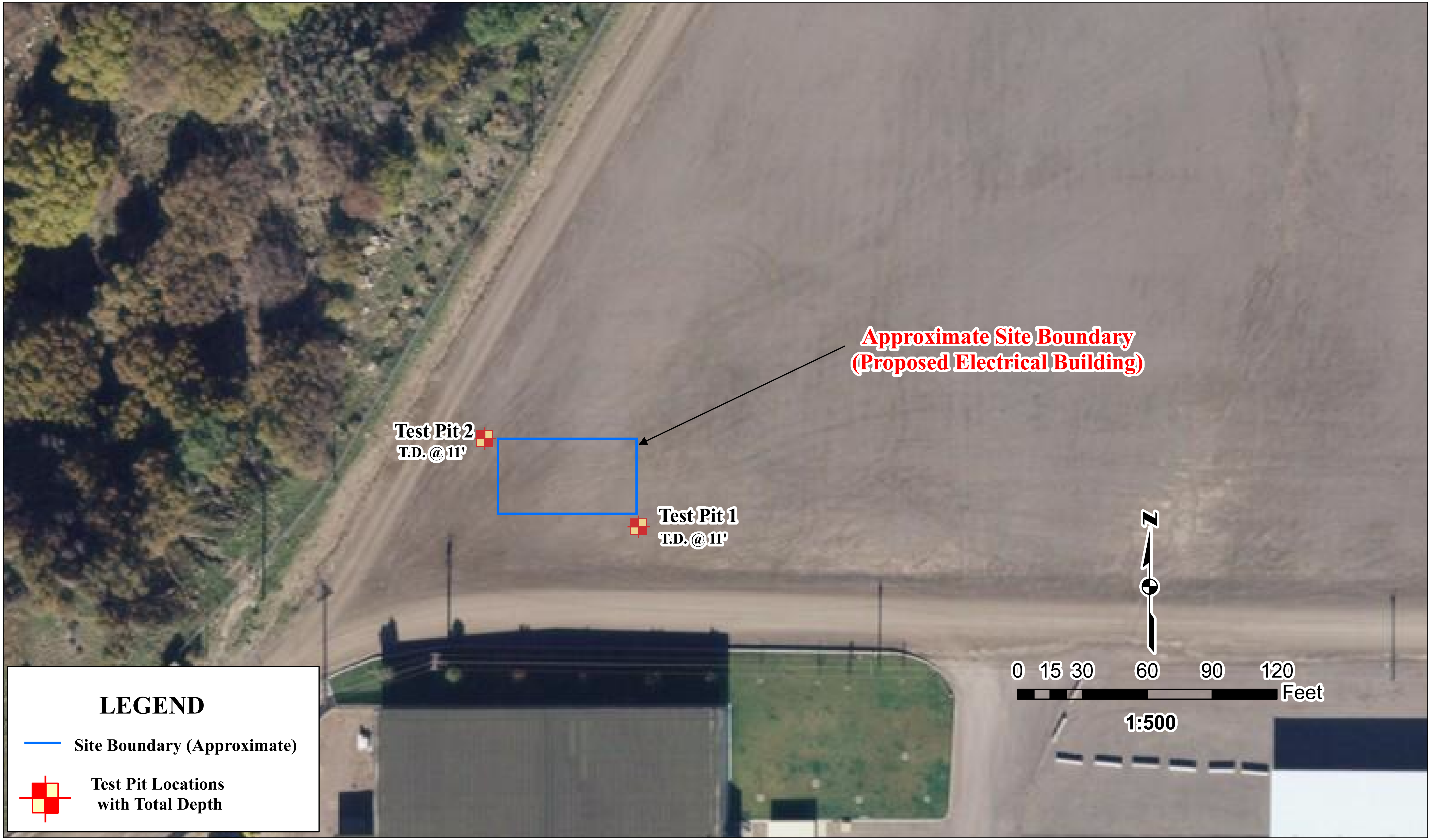
Project Number - 01273-002

Geotechnical Investigation
Camp Williams
17800 South Camp Williams Rd.
Riverton, Utah

SITE VICINITY MAP

Plate

A-1



IGES

Project Number - 01273-002

Geotechnical Investigation
Camp Williams
17800 South Camp Williams Rd.
Riverton, Utah 84065

GEOTECHNICAL MAP

Plate
A-2

DATE			Geotechnical Investigation Camp Williams 17800 South Camp Williams Rd. Riverton, Utah Project Number 01273-002	IGES Rep: CLE Rig Type: Case 480 Backhoe	TEST PIT NO: **TP-1** Sheet 1 of 1
STARTED:	6/16/09				
COMPLETED:	6/16/09				
BACKFILLED:	6/16/09				

LOCATION: NORTHING | EASTING | ELEVATION

DEPTH (METERS)	DEPTH (FEET)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0	CL	Undocumented FILL - Lean CLAY with gravel - hard, slightly moist, brown, fine-grained, gravel 1/2" to 1' gravel					
					7.9			
		GP	Poorly Graded GRAVEL with sand - dense, slightly moist, tan-brown, cemented with calcium carbonate, contains rounded to subrounded clasts 1 to 1.5" in diameter - Lake Bonneville sediments					
1								
	5				4.6	7.8		
					5.4			
2								
3	10							
			No Groundwater Encountered					
			Bottom of Test Pit @ 11 Feet					

Moisture Content and Atterberg Limits: Plastic Limit — Moisture Content — Liquid Limit; 10 20 30 40 50 60 70 80 90

IGES

SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate

A - 3

DATE	
STARTED:	6/16/09
COMPLETED:	6/16/09
BACKFILLED:	6/16/09

Geotechnical Investigation
Camp Williams
17800 South Camp Williams Rd.
Riverton, Utah

Project Number 01273-002

IGES Rep: CLE
Rig Type: Case 480 Backhoe

TEST PIT NO: **TP-2**
Sheet 1 of 1

LOCATION
NORTHING | EASTING | ELEVATION

DEPTH (METERS)	DEPTH (FEET)	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0	0	CL	Undocumented FILL - Lean CLAY - hard, slightly moist, brown, 1/2 to 1" gravel					
		GP	Poorly graded GRAVEL - loose, slightly moist, gray-brown, clasts are subrounded to rounded 1" to 1' in diameter					
1		ML	Sandy SILT with trace gravel - stiff, slightly moist, tan, horizontal calcium carbonate veins, crude bedding	83.8	22.3		NP	NP
	5	SP	Poorly Graded SAND - medium dense, slightly moist, tan, coarse-grained, bedded					
2		SM	Silty SAND with trace gravel - medium dense, slightly moist, tan, calcium carbonate veins, bedded, subrounded to rounded clasts of andesite gravel		15.0	34.7		
3	10	SP	Poorly Graded SAND - medium dense, slightly moist, brown-gray, coarse-grained, bedded		7.2	4.3		
			No Groundwater Encountered					
			Bottom of Test Pit @ 11 Feet					

Moisture Content and Atterberg Limits
Plastic Limit — Moisture Content — Liquid Limit
10 20 30 40 50 60 70 80 90

SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Plate
A - 4

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKELY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SU	SOLUBLE SULFATES
COMP	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"
LAYER	1/2 - 12"

DESCRIPTION	THICKNESS
OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT (blows/ft)	TORVANE UNTRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

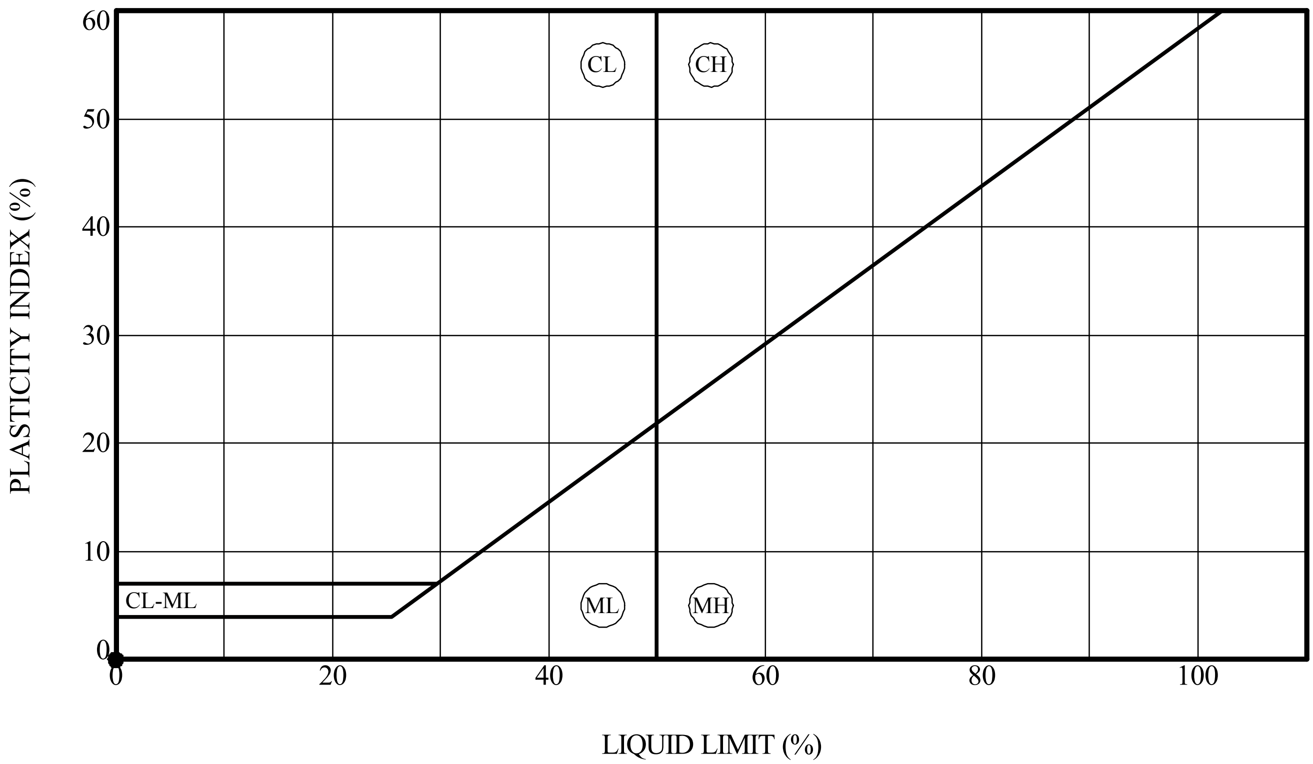
IGES

Key to Soil Symbols and Terminology

Plate A-5

IGES, Inc. Project No.:01273-002

APPENDIX B



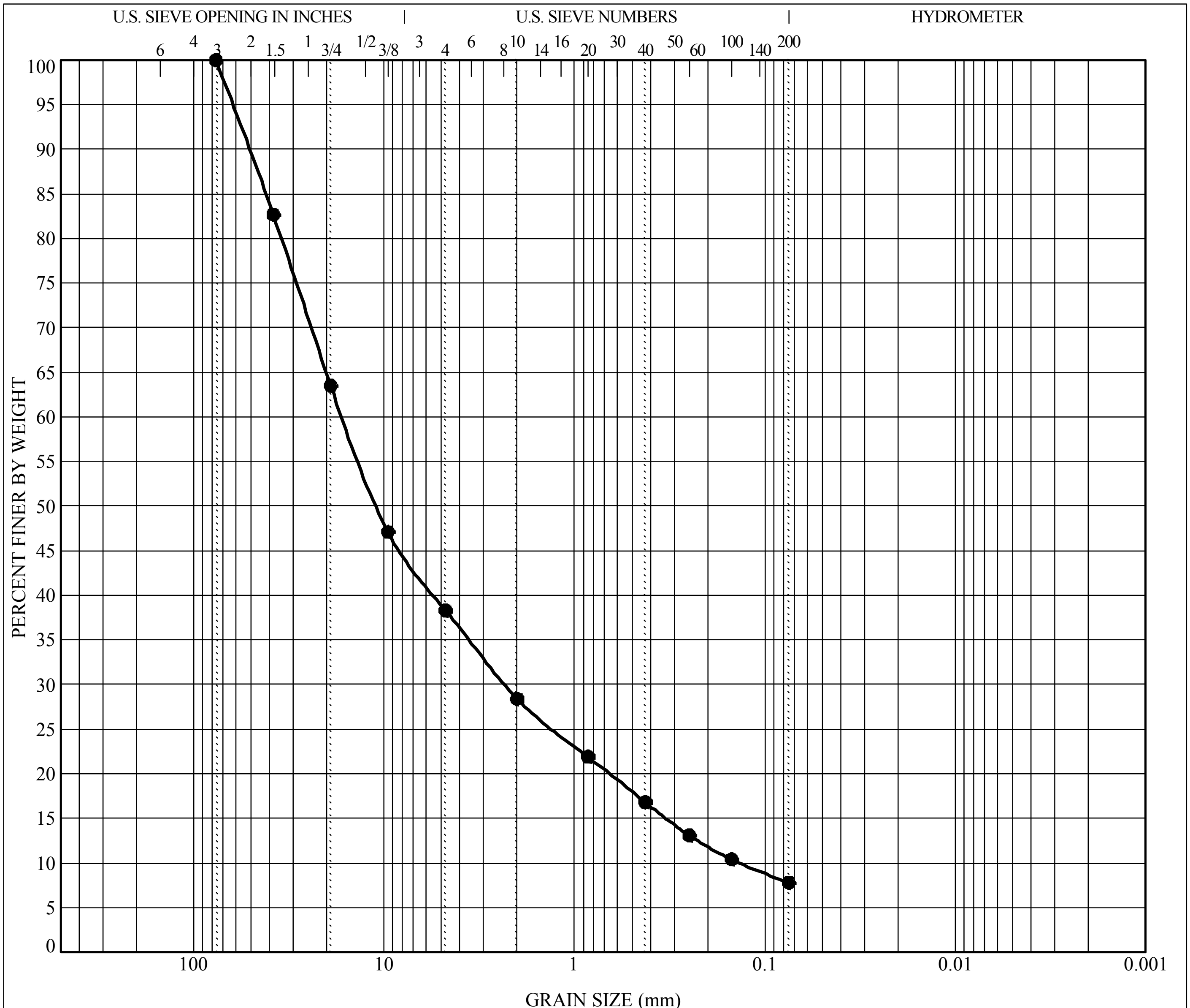
	Sample Location	Depth (ft)	LL (%)	PL (%)	PI (%)	Classification
●	**TP-2**	**3.5**	**NP**	**NP**	**NP**	**Sandy SILT with trace gravel (ML)**

ATTERBERG LIMITS' RESULTS

IGES

Geotechnical Investigation
Camp Williams
17800 South Camp Williams Rd.
Riverton, Utah
Project Number: 01273-002

Plate **B - 1**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	Depth	Classification	LL	PL	PI	Cc	Cu
●	**TP-1**	**5.0**	**Poorly Graded GRAVEL with sand (GP)**				**2.39**	**121.86**

	Sample Loctaion	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP-1**	**5.0**	**76.2**	**16.43**	**2.3**	**0.135**	**61.7**	**30.5**	**7.8**	

IGES

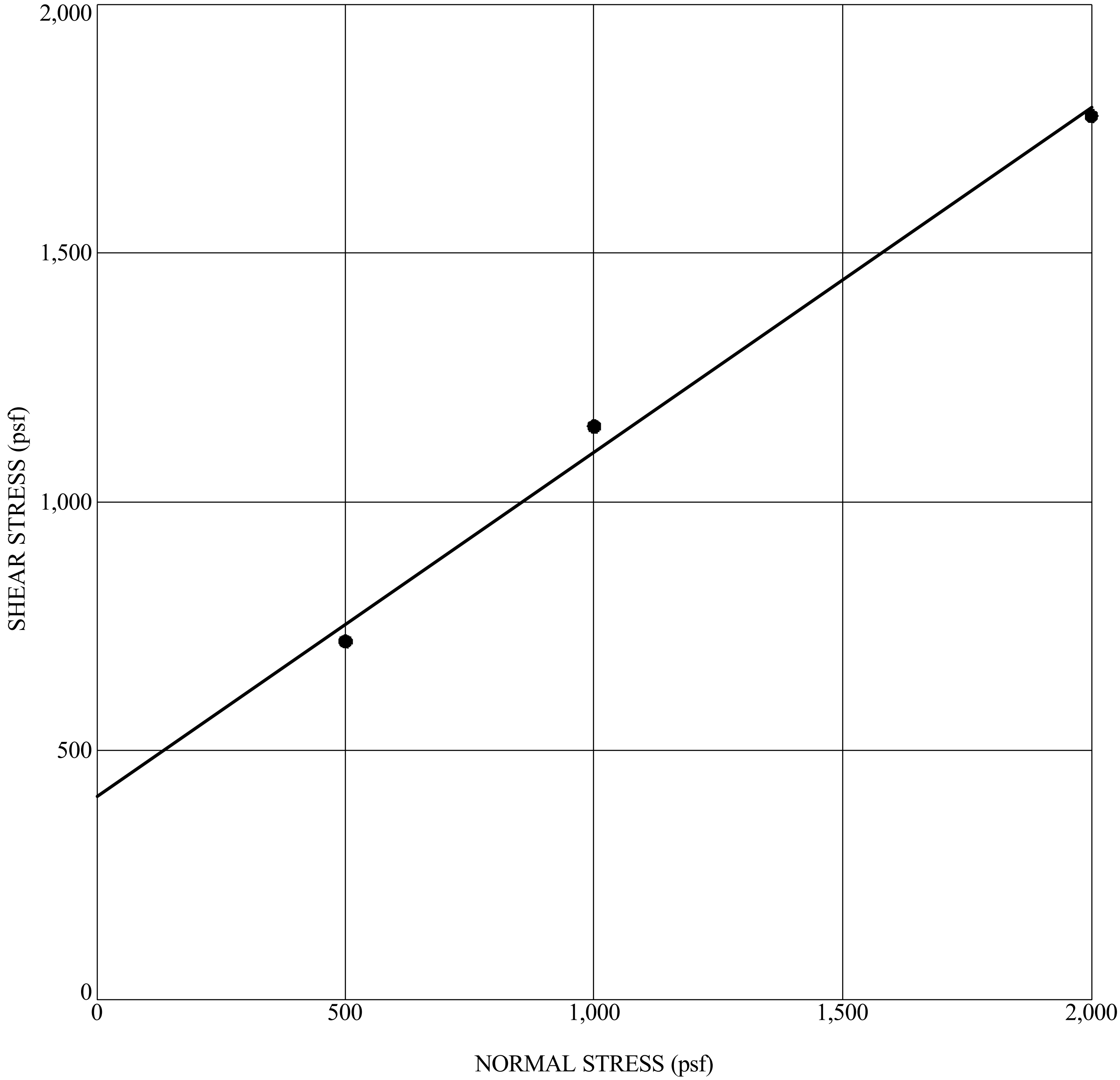
GRAIN SIZE DISTRIBUTION

Geotechnical Investigation
Camp Williams
17800 South Camp Williams Rd.
Riverton, Utah
Project Number: 01273-002

Plate

B - 2

B GSD CAMP WILLIAMS 01273-002.GPJ IGES.GDT 6/25/09



	Sample Location	Depth (ft)	Classification	γ_d (pcf)	MC (%)	c (psf)	ϕ (deg)	
●	**TP-2**	**3.5**	**Sandy SILT with trace gravel (ML)**	**83.8**	**22**	**408**	**35**	

IGES

DIRECT SHEAR TEST

Geotechnical Investigation
Camp Williams
17800 South Camp Williams Rd.
Riverton, Utah
Project Number: 01273-002

Plate

B - 3

B_DIRECT_SHEAR CAMP WILLIAMS 01273-002.GPJ IGES.GDT 6/25/09

SUMMARY OF LABORATORY TEST RESULTS TABLE

Geotechnical Investigation **17800 South Camp Williams Rd., Riverton, Utah** **Project Number: 01273-001**

SAMPLE LOCATION		NATURAL DRY DENSITY (pcf)	NATURAL MOISTURE CONTENT %	GRADATION (%)			ATTERBERG LIMITS		DIRECT SHEAR		CHEMICAL TESTS				UNIFIED SOILS CLASSIFICATION
Point No.	Depth (ft)			Gravel >#4	Sand	Silt and Clay <#200	Liquid Limit	Plasticity Index	C (psf)	Ø	Soluable Chloride (ppm)	Soluable Sulfate (ppm)	Resistivity (Minimum ohm-cm)	pH	
TP-1	1.5		7.9												Fill - Lean CLAY with gravel (CL)
	5		4.6	61.7	30.5	7.8									Poorly Graded GRAVEL with sand (GP)
	6		5.4												Poorly Graded GRAVEL with sand (GP)
TP-2	3.5	83.8	22.3				NP	NP	408	35	760	41	388	7.9	Sandy SILT with trace gravel (ML)
	7		15			34.7									Silty SAND with trace gravel (SM)
	10		7.2			4.3									Poorly Graded SAND (SP)

Plate B-4

APPENDIX C

SITE GROUND MOTION [IBC SECTION 1613]

Project:	**Camp Williams**	**Number:**	**01273-002**
Latitude =	**40.4383**	**Date:**	**6/19/09**
Logitude =	**-111.9283**	**By:**	**CLE**

S_s = 1.122 (g) The mapped spectral accleration for short periods [1613.5]

S_1 = 0.457 (g) The mapped spectral accleration for a 1-second period

Site Class = D Table 16.13.5.2

Fa = 1.05 Table 1613.5.3(1)

Fv = 1.54 Table 1613.5.3(2)

S_{MS} = 1.179 $S_{MS} = Fa*Ss$ *The maximum considered E.Q. spectral resonse accelerations

S_{M1} = 0.705 $S_{M1} = Fv*S_1$ for short and 1-second periods [1613.5.3]

MCE/PGA = 0.472 **$0.4*S_{MS}$ [In accordance with 1802.2.7]**

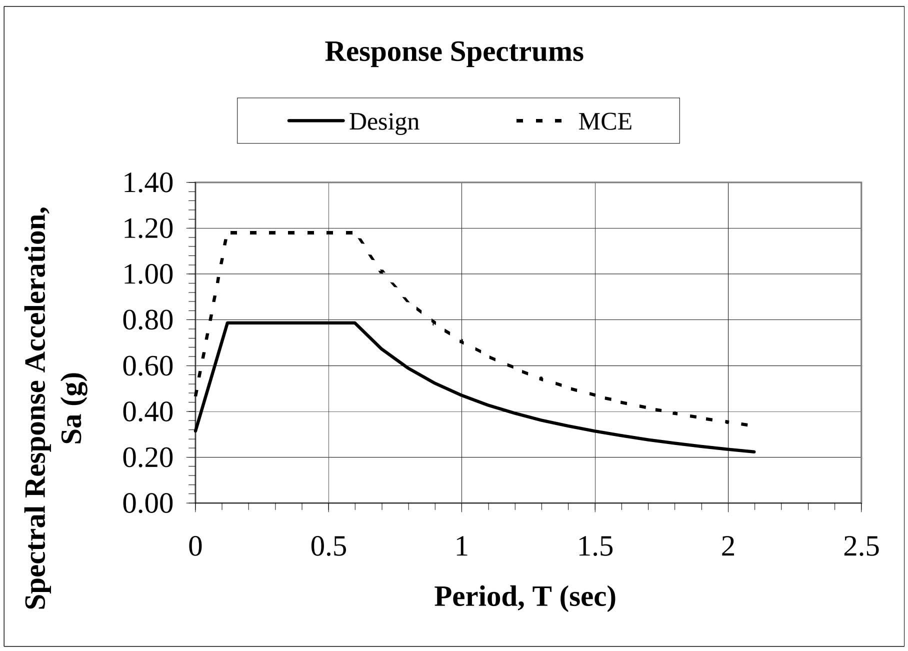
S_{DS} = 0.786 $S_{DS} = 2/3*S_{MS}$ *The design spectral response acceleration

S_{D1} = 0.470 $S_{D1} = 2/3*S_{M1}$ at short and 1-second periods

T_0 = 0.120 $T_0 = 0.2*S_{D1}/S_{DS}$

T_s = 0.598 $T_s = S_{D1}/S_{DS}$ C

ΔT = 0.1 Time step for diagram



T (sec)	Sa (g)	Sa (MCE) (g)
0	0.31	0.47
0.12	0.79	1.18
0.60	0.79	1.18
0.70	0.67	1.01
0.80	0.59	0.88
0.90	0.52	0.79
1.00	0.47	0.71
1.10	0.43	0.64
1.20	0.39	0.59
1.30	0.36	0.54
1.40	0.34	0.50
1.50	0.31	0.47
1.60	0.29	0.44
1.70	0.28	0.42
1.80	0.26	0.39
1.90	0.25	0.37
2.00	0.24	0.35
2.10	0.22	0.34

SUMMARY OF GEOLOGIC HAZARDS

Camp Williams Springville, Utah **Project Number 01273-002**

Hazard	Hazard Rating*				Further Study Recommended**
	Not Assessed	Probable	Possible	Unlikely	
Earthquake					
Ground Shaking		X			See Geotechnical Report
Surface Faulting				X	
Tectonic Subsidence				X	
Liquefaction	X		X		See Geotechnical Report
Slope Stability				X	
Flooding (Including Seiche)				X	
Slope Failure					
Rock Fall				X	
Landslide				X	
Debris Flow				X	
Avalanche				X	
Problem Soils					
Collapsible				X	
Soluble				X	
Expansive				X	
Organic				X	
Piping				X	
Non-Engineered Fill		X			See Geotechnical Report
Erosion				X	
Active Sand Dune				X	
Mine Subsidence				X	
Shallow Bedrock				X	
Shallow Groundwater				X	
Flooding					
Streams				X	
Alluvial Fans				X	
Lakes				X	
Dam Failure				X	
Canals/Ditches				X	
Radon	X				

* Hazard Rating :
Not assessed - report does not consider this hazard and no inference is made as to the presence or absence of the hazard at the site
Probable -Evidence is strong that the hazard exists and mitigation measures should be taken
Possible - hazard may exist, but the evidence is equivocal, based only on theoretical studies, or was not observed and furthes study is necessary as noted
Unlikely - no evidence was found to indicate that the hazard is present, hazard not known or suspected to be present

Further Study :
E - geotechnical/engineering, H - hydrologic, A - Avalanche, G - Additional detailed geologic hazard study out of the scope of this study